STEEL AND CONCRETE SUBSTRUCTURE
OF A
RIVER CROSSING FOR HIGHWAY TRAFFIC

BY

J. A. HOLMBOE

ARMOUR INSTITUTE OF TECHNOLOGY

1914

DESIGN AND ESTIMATE OF COST OF
STEEL SUPERSTRUCTURE AND CONCRETE SUBSTRUCTURE OF A
RIVER CROSSING FOR HIGHWAY TRAFFIC.

A THESIS

Presented by
J. A. Holmboe

To The
PRESIDENT AND FACULTY
Of
ARMOUR INSTITUTE OF TECHNOLOGY

For The Degree Of
BACHELOR OF SCIENCE IN CIVIL ENGINEERING

Having Completed The Prescribed
Course Of Study In
CIVIL ENGINEERING

1914.

Approved
A. E. Phillips
Prof Civil Engineering
H. M. Raymond
L. C. Monin

The Problem.

The problem consists of the design of a river crossing for highway traffic for a suburb of Chicago. The location and general conditions are assumed as shown on the blue print. The river is bridged by three spans, the middle span being seventy five feet and the two end spans each sixty feet. There will be two solid concrete piers and two reinforced concrete abutments.

List of Illustrations.

Table of Contents.

Design of Middle Span.

Span = 75' - 0".

Weight = 8' - 0"

Width center to center of Trusses = 18' - 6".

Pratt Trusses, parallel chords, no sidewalks.

Asphalt floor with concrete foundation.

Specifications - Illinois Highway Commission.

Class "C" bridge.

$\frac{3}{4}$" rivets.

Design of Floor.

Assume concrete foundation 4" thick. (FIG. 2)

Buckle plates - 5/16" thick.

Binder course - $1\frac{1}{2}$" thick.

Asphalt - 3" thick.

Total weight of floor taken as 160 # per cu.ft.

Weight of floor = 105 # per square foot.

Weight of floor per panel = 15 x 17 x 105 = 26775 #

Heavier wheel load = 4 tons = 8000 #.

Space the stringers 2' - $1\frac{1}{2}$" apart.

Design of Roadway Stringers.

To find the center of gravity:-

$\frac{8000 \times 10}{12000} = 6.667$ (fig. 3)

For the maximum bending moment, the heaviest load must be upon the span and a wheel load must be at the section and this wheel load causing the maximum bending moment must be as far on one side of the center as the center of gravity of all the loads is upon the other. In this case however the maximum bending moment is caused by a wheel load placed at the center of the span.

$\frac{10 - 6.667}{2} = 1.667'$ distance of center of stringer to center of wheel # 1. This makes wheel #2 come off the span. There put wheel #8 at the center of the span.

$M = \frac{8000}{2} \times 7.5 \times 12 = 360000$ "#

$\frac{360000}{13000} = 27.7$ Section Modulus.

Carnegie :- Try 12" I-beam @ 27.5 # $S = 33.3$

Weight of floor = 105 # per foot.

Weight of stringer = 27.5 # per foot.

Total weight = 132 # per foot. D.L.

$D.L.B.M. = \frac{wl^2}{8} = \frac{132.5 \times 15^2 \times 12}{8} = 44800$ "#

$\frac{44800}{13000} = 3.44$

$27.7 + 3.44 = 31.14$ O.K. since 33.3 is greater than 31.14

Design of Track Stringers.

Live load is 18 tons on two axles 10 foot centers. (Fig.4)

$$L.L.B.M. = \frac{9000}{2} \times 7.5 \times 12 = 405000 \text{ "\#}$$

$$S = \frac{405000}{13000} = 31.2$$

Try 12" I-beam @ 31.5 # $S = 36.0$

D.L.	105.0 #	
	31.5	
	40.0	I-beam
	30.0	Rail
	206.5 #	Total.

$$M = \frac{wl^2}{8} = \frac{215 \times 15^2 \times 12}{8} = 69800 \text{ "\#}$$

$69800 + 361200 = 431000$

$$S = \frac{431000}{13000} = 33.2 \quad \text{O.K.}$$

(Art.57) $\frac{75}{20}$ is less than 12"

Design of Floor Beams.

Assume concentrated loads are supported by two stringers, the maximum shear occurs when

one axle at the end of the panel.

Weight of the concrete floor = 105 x 15 x 17.33

= 27250 #

2-12" I-beams	2 x 15 x 31.5 =	945 #
7-12" I-beams	7 x 15 x 27.5 =	3680
Rails 2 x 30 x 15 =		900
		5525
Add 15% for details =		828
	Total steel =	6350
	Floor =	27250
		33603
Assume weight of floor beam =		1400
Totoal Dead Load.----------		35003 #

$$D.L.B.M. = \frac{wl}{8} = \frac{35003 \times 17.5 \times 12}{8} = 917500 \text{ "\#}$$

Live Load Stresses.

There is a chance for two kinds of loading:-

(1) Street car and uniform Load. Use this for moments. (Fig.5)

(2) Traction Engine and uniform load. Use this for shear. (Fig.6)

9000 + 9000 x 5/15 = 9000 + 3000 = 12000'# = 144000"#

Moment of street car.

Middle 12' width = 10 x 100 x 5/15 = 334 # per foot floor beam.

Condition od live load for maximum moment :-

Reaction = 2 x 1500 = 3000

$6 \times 334 = 1944$

Car load = $\underline{12000}$

16944

Moments at the center = $16944 \times \frac{17.50}{2} - 3000 \times 7$

$- 1944 \times 3 - 12000 \times 2\frac{1}{2} = 148300 - 21000 - 5832$

$- 25000 = 148300 - 51830 = 96470$ '# = 1156000 "#.

Total moment = 1156000 + 917500 = 2073500 "# = 2073.5 inch kips.

Section Mod. = $\frac{2073.5}{13} = 159.6$

Use 24" @ 80# I-beam   S = 173.9   O.K. for B.M.

For shear set the traction engine against the curb. Traction engine is assumed to cover 8' x 14'. Uniform load as before. (Fig. 6)

Uniform load at the ends of traction engine:-

$\frac{3 \times 100 \times 1.5}{15} + \frac{13 \times 100 \times 6.5}{15} = 594$ #

Left reaction = $\frac{594 \times 8 \times 13.33}{18.66} + \frac{1500 \times 9 \times 5.33}{18.66}$

$+ \frac{12000 \times 13.33}{18.66} = 3400 + 3860 + 8570 = 15850$ #

100 # per sq. ft. alone gives $\frac{16 \times 15}{2} \times 100$

$= 12000$

The other loading = 1694

Area of section = 23.32 sq.in which is more than enough foe shear.

Maximum Stringer Reaction.

Roadway Stringers:- (Fig.7)

$R = 8000 + 5/15 \times 4000 = 9333$ #

$\frac{3 \times 2 \times 100 \times 1.5}{15} = 60$

D.L. $= 140 \times 15 = \underline{2100}$

Total $= 9333 + 60 + 2100 = 11493$ #

Track Stringers.

$R = 9000 + 3000 =$	12000
Dead Load =	206.5
No uniform live load.	12206.5 Total

Connection Angles on Stringers.

$\frac{3}{4}$" rivets

Art. 53. $10000 \times 80\% = 8000$ # shear.

$20000 \times 80\% = 16000$ # bearing.

Single shear = 3530

Double Shear = 7070

Bearing Values of Rivets.

$\frac{1}{4}$	5/16	3/8	7/16	$\frac{1}{2}$	9/16
3000	3750	4500	5250	6000	6750

Roadway Stringers.

Lines AA and BB in bearing and double shear.

$\frac{11493}{3750} = 4$ rivets req'd. (Fig.6)

Rivets in floor beam are in single shear or bearing on web of floor beam.

$\frac{11493}{3530} = 4$ shop rivets or 6 field rivets.

Track Stringers.

Web = 3/8"

$\frac{12206.5}{4500} = 3$ rivets

Web of floor beam $= \frac{12206.5}{3530} = 4$ rivets or 6 field.

Use 12" I-beam.

Use 2 angles 4" x 4" x 7/16" x 0'-8½" Standard.

Floor Beam Connection To Post.

In web of floor beam the rivets are in double shear or bearing.

$\frac{15850}{6000} = 3$ rivets

In post assume single shear, since post should not be thinner than 5/16".

$\frac{15850}{3520} = 5$ shop rivets or 8 field rivets.

Use 2 angles 6" x 4" x ½" x 1'-5½" long. St'd.

Loads For Trusses.

1130 # per foot of car track.

73 # per foot of remaining surface.

Per panel per truss L.L.

$1130 \times 7\frac{1}{2} = 8480$

$73 \times 7\frac{1}{2} \times (17.5 - 10) = 4110$

Sum $= 8480 + 4110 = 12590$ # L.L.

Per panel per truss. D.L.

Floor $= 17.5 \times 7\frac{1}{2} \times 105 = 13800$

Rails $= 50 \times 15 = 750$

Total $= 14550$

Steel in floor $= \frac{15}{2} \times 80 = 600$ #

Roadway stringer $= 3\frac{1}{2} \times 35 \times 15 = 1841$

Track stringer $= 1 \times 31.5 \times 15 = 472$ #

Sum $= 600 + 1841 + 472 = 2913$ #

10 lineal feet 6" x 4" x 3/8" angles @ 12#

$= 120$ #

$120 + 2913 = 3033$ say 3050 #.

This makes 1175 # per lineal foot of bridge

per truss. Railing = 30# per lineal foot or

450 # per panel per truss.

Steel from Du Four's formula

$\frac{15}{2}(350 + 3.5 \times 75) = 3260$ # per panel per truss.

$3265 + 13800 = 17065$ per panel per truss or say 17100 #

Stresses.

$\tan\theta = \frac{8}{7.5} = 1.06666 \quad \theta = 46^\circ - 50' \quad \sec\theta = 1.46173$

D.L. = w = 17100 #

Chord eg. c. og m. at F.

$$\frac{2w \times 2\frac{1}{2}p - w(1\frac{1}{2} + \frac{1}{2})p}{h} = \frac{5wp - 2wp}{h}$$

$$= \frac{3 \times 17100 \times 15}{8} = 96000 \#$$

Chord DF. c. of m. at e.

$$\frac{2w \times 2p - wp}{h} = \frac{3wp}{h} = \frac{3 \times 17100 \times 15}{8} = 96000$$

Chord ce. c. of m. at D.

$$\frac{2w \times 1\frac{1}{2}p - w \times \frac{1}{2}p}{h} = \frac{3wp - \frac{1}{2}wp}{h} = \frac{2\frac{1}{2}wp}{h}$$

$$= \frac{2\frac{1}{2} \times 17100 \times 15}{8} = 80200$$

Chord BD. c. of m. at c.

$$\frac{2wp}{h} = \frac{2 \times 17100 \times 15}{8} = 64100 \#$$

Chord Ac. c. of m. at B.

$$\frac{2w \times \frac{1}{2}p}{h} = \frac{wp}{h} = \frac{17100 \times 15}{8} = 32100 \#$$

Diagonals.

Shear in panel EG = 0.

Therefore there is no dead load stress in the diagonals.

Shear in panel CE = 2w - w = w = 17100

Stress = 17100 secθ = 25000 # = De = -Dc

Shear in panel AC = 2w = 34200

Stress = 34200 sec θ = 50000 # = Bc = -AB.

Posts.

Post Cc.

Dc sin θ + Cc + Bc sin θ = 0

-25000 sin θ + Cc + 50000 sin θ = 0

Cc = -(50000-25000) sin θ = -18250 #

Post Ee.

Fe sin θ + Ee + De sin θ = 0

0 + Ee = -25000 sin θ = -18250 #

Live Load Stresses.

Ratio = $\frac{12590}{17100}$ = 0.74

eg = DF = 71000

ce = 59400

BD = 47400

Ac = 23800

$Bc = \frac{(1+2+3+4)w'}{5} \sec\theta = 10/5 \times 12590 \times 1.46173$

$= 36810$ #

$AB = -Bc = -36810$

$De = \frac{(1+2+3)w'}{5} \sec\theta = 6/5 \times 12590 \times 1.46173$

$= 22100$

$Dc = -De = -22100$

$Fg = -Fe = \frac{(1+2)w'}{5} \sec\theta = 3/5 \times 12590 \times 1.46173$

$= 11050$ #

Cc.

$Dc \sin\theta + Cc + Bc \sin\theta = 0$

$-22100 \sin\theta + Cc + 36810 \sin\theta = 0$

$Cc = -(36810 - 22100) \sin\theta = -14710 \times .72937$

$= -10720$

Ee.

$Fe \sin\theta + Ee + De \sin\theta = 0$

$-11050 \sin\theta + Ee + 22100 \sin\theta = 0$

$Ee = -(22100 - 11050) \sin\theta = -11050 \sin\theta = -8060$ #

Design of Upper Chord.

DF.

D.L.		96000
L.L.		71000
Impact L.L.	$\frac{300}{300+45}$	56800
		223800

$S = 18000 - 70\ l/r$

Assume section as follows:-

1 Cover Plate	12" x ½" =	6.00
2 angles 6" x 4" x 5/8" =		11.72
		17.72 sq. in.

To find the center of gravity of the section take moments about the top.

$$y = \frac{6 \times .25 + 11.72 \times 2.53}{17.72} = \frac{1.5 + 29.61}{17.72} = 1.75\,"$$

Moments of inertia about the center of gravity.

$$I_a = 1/12 \times 12 \times (.5)^3 + 6.00 \times (1.5)^2 = 13.63$$

$$I_b = 2 \times 21.1 + 11.72 \times (.72)^2 = \underline{48.28}$$

$$61.91$$

$$r = \sqrt{I/a} = \sqrt{\frac{61.91}{17.72}} = 1.85$$

$$S = 18000 - \frac{70 \times 7.5 \times 12}{1.85} = 18000 - 3400 = 14000$$

Take S = 14000 in designing.

$$\frac{223800}{14000} = 15.9 \text{ sq. in.}$$

$$\text{Rivets req'd.} = \frac{17.72 \times 14000}{8000} = 31 \text{ shop or } 47 \text{ field}$$

	FD.
D.L.	64100
L.L.	47400
Impact	38400
	149900 Total

Use the same section as for DF.

Design of End Post.

AB.

Use the same section as for DF here also.

Length $= \sqrt{8^2 + 7.5^2} = \sqrt{120.2} = 10.96$

D.L.	50000
L.L.	36810
Impact	29400
	116210

$S = 18000 - 80\ l/r = 18000 - \frac{80 \times 10.96 \times 12}{1.85}$

$= 12300$

Area req'd $= \frac{116210}{12300} = 9.5$ sq. in. Sect. O.K.

31 rivets req'd as in upper chord.

Design of Lower Chord Tension Members.

Lower Chord eg.

D.L.	96000
L.L.	71000
Impact $\frac{300}{300 + 75}$	56800
	223800

Net area req'd $= \frac{223800}{16000} = 14.0$ sq. in.

Try 2 angles 8" x 4" x 7/8" Gross area $= 15.96$

Deduct 2 rivet holes @ .77 $=$ 1.54

14.42

Rivets req'd in lower chord eg:-

$\frac{14.42 \times 16000}{8000} = 29$ shop rivets or 44 field

Lower Chord ce.

D.L.		80200
L.L.		59400
Impact	$\frac{300}{300 + 75}$	47600
		187200

Net area req'd $= \frac{187200}{16000} = 11.7$ sq. in.

Try 2 angles $6" \times 4" \times \frac{3}{4}" =$	13.88 sq. in.
Deduct 2 rivet holes @ .66 =	1.32
	12.56 sq.in.

$\frac{12.56 \times 16000}{8000} = 26$ shop rivets or 39 field .

Design of Post.

Cc.

D.L.		18252
L.L.		10720
Impact L.L.	$\frac{300}{300 + 60}$	8900
		37870 Compression.

$S = 16000 - 80\ l/r$

Try 2 angles $4" \times 3" \times 5/16"$ Area = 4.18 sq.in.

$I = 2 \times 3.4 = 6.8$

$r = \sqrt{I/a} = \sqrt{\frac{6.8}{4.18}} = \sqrt{1.63} = 1.28$

$S = 16000 - \frac{80 \times 8 \times 12}{1.28} = 10000$

Area req'd $= \frac{37870}{10000} = 3.79$ sq.in Sect. O.K.

Rivets req'd $= \frac{4.18 \times 10000}{8000} = 6$

Design of Diagonals.

Bc--Tension Member.

D.L.		50000
L.L.		36810
Impact L.L.	$\frac{300}{300 + 60}$	30600
		117410

Net area req'd $= \frac{117410}{16000} = 7.35$ sq. in.

Try 2 angles 4" x 3" x 11/16" Gross area = 8.68

Deduct 2 rivet holes @ .60 = 1.20

Net area = 7.48

Rivets req'd $= \frac{7.48 \times 16000}{8000} = 15$ rivets

De--Tension Member.

D.L.		25000
L.L.		22100
Impact L.L.	$\frac{300}{300 + 60}$	18450
		65550

Net area req'd $= \frac{65550}{16000} = 4.1$ sq. in.

Try 2 angles 4" x 3" x ½" = 6.50 sq.in.

Deduct 2 rivet holes @ .44 = .88

5.62 O.K.

Diagonals - Compression

Diagonals - Compression.

$Dc = -De = 65550$

$S = 16000 - 80\ l/r$

Try 2 angles 4" x $3\frac{1}{2}$" x $\frac{3}{4}$"   A = 10.12 sq.in.

$I = 2 \times 7.3 = 14.6$

$r = \sqrt{I/a} = \sqrt{\frac{14.6}{10.12}} = 1.20$

$S = 16000 - \frac{80 \times 11 \times 12}{1.20} = 7200$

$\frac{65550}{7200} = 9.12$ sq. in.   O.K.

Rivets req'd $= \frac{10.12 \times 7200}{6000} = 10$ shop or 15 field.

Design of Diagonals in Middle Panel.

Fe (compression) and Fg (tension) = 11050.

Live Load Stresses Only.

		Fe.
L.L.		11050
Impact L.L.	$\frac{300}{300 + 30}$	10500
		21550

$S = 16000 - 80\ l/r$

Try 2 angles 3" x 3" x $\frac{1}{2}$"   Area = 5.50 sq.in.

$I = 2 \times 2.2\quad 4.4$

$r = \sqrt{I/a} = \sqrt{\frac{4.4}{5.5}} = .895$

$S = 16000 - \frac{80 \times 11 \times 12}{.895} = 4200$

$\frac{21550}{4200} = 5.13$ sq. in. req'd O.K.

Rivets req'd $= \frac{5.5 \times 4200}{8000} = 3$ rivets

Lateral Bracing.

(Fig. 9) According to specifications the wind load is 300 pounds per lineal foot dead load and 150 pounds per lineal foot live load. The panel Dead Load is 15 x 300 = 4500 #.

$\text{Tan } \theta = \frac{18.5}{15} = 1.233$

$\theta = 50^\circ - 50'$

$\sec \theta = 1.58333$

L.L. per panel = 15 x 150 = 2250

Dead Load Stresses.

Left reaction = $2\frac{1}{2}$ x 4500 = 11250

Shear in panel EG for D.L. = 0

Therefore no dead load stresses in diagonals.

Shear in panel CE.

Shear $= (2\frac{1}{2} - \frac{1}{2} - 1)w = w = 4500$

Stress in diagonals = 4500 sec θ = 7120

Shear in panel AC.

Shear $= (2\frac{1}{2} - \frac{1}{2})w = 2w = 9000$

Stress in diagonals = 9000 sec θ = 14240

Live Load Stresses.

Panel EG. Live Load up to g.

$$\text{Shear} = \frac{(1+2)w}{5} = 3/5w = 3/5 \times 2250 = 1350$$

Stress in diagonals = 1350 sec θ = 2140 #

Panel CE. Live load up to e.

$$\text{Shear} = \frac{(1+2+3)w}{5} = 6/5 \times 2250 = 2700$$

Stress in diagonals = 2700 sec θ = 4275

Panel AC. Live load up to c.

$$\text{Shear} \quad \frac{(1+2+3+4)w}{5} = 2w = 2 \times 2250 = 4500$$

Stress in diagonals = 4500 sec θ = 7120

Total stress in Eg = 2140

$$\text{Area req'd} = \frac{2140}{16000} = .134$$

Try one angle $3\frac{1}{2}$" x 3" x 3/8"   Area = 2.3 sq.in.

Net area = 2.3 - .38 = 1.92 sq.in.

$$\text{Shop rivets req'd} = \frac{1.92 \times 16}{8} = 4 \quad \text{or 6 field.}$$

Ce

L.L.	4225
D.L.	7120
	11345

$$\text{Area req'd} \quad \frac{11345}{16000} = .71 \text{ sq.in.}$$

Try 1 angle $3\frac{1}{2}$" x 3" x 3/8"   Area = 2.3 sq.in.

Net area = 2.30 - .38 = 1.92 sq.in.

4 shop rivets or 6 field.

Ac.

D.L.	14240
L.L.	7120
	21360 Total

Area req'd $\frac{21360}{16000} = 1.33$ sq. in

Try 1 angle $3\frac{1}{2}$" x 3" x 3/8"   Area = 2.30 sq.in.

Net area = 2.30 - .38 = 1.92 sq.in.

4 shop rivets or 6 field.

Design of End Span.

The dimensions of this bridge are the same as for the middle span except that the length is sixty feet instead of seventy five feet. There will be four panels at fifteen feet and the height will be eight feet as was the case in the preceding design.

Design of Floor.

The design of the floor will be the same in this case as in the preceeding design.

Design of Stringers.

Since the panel length is the same for both bridges, the design of the stringers will be the same.

Design of Floorbeam.

The floorbeams will be the same for both bridges. The connection angles on the stringers and floorbeams will also be the same.

Loads for Trusses.

The weight of the floor per panel will be the same or 13800 #

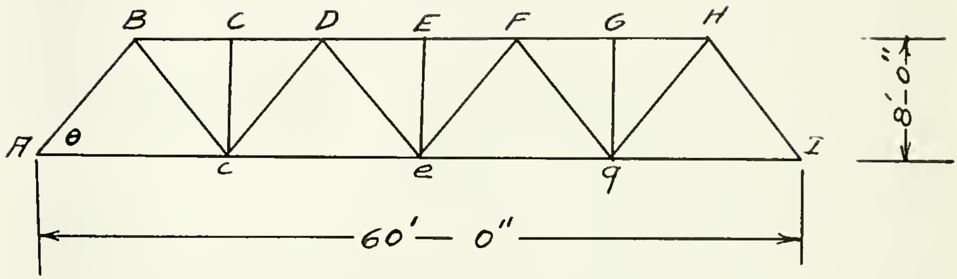
Steel from Du Four's formula:-

$\frac{15}{2}(250 + 2.5 \times 60) = 3000$ #

13800 + 3000 = 16800 # per panel per truss D.L.

The live load per panel per truss will be the same in both cases = 12590 #

Stresses.



$\theta = 46^\circ - 50'$ sec $\theta = 1.46173$

D.L. = w = 16800 #

Chord DF c. of m. at e.

$$\frac{1\frac{1}{2}w \times 2p - wp}{n} = \frac{2wp}{n} = \frac{2 \times 16800 \times 15}{8} = 63000$$

Chord ce. c. of m. at D.

$$\frac{1\frac{1}{2}w \times 1\frac{1}{2}p - w \times \frac{1}{2}p}{n} = \frac{2\frac{1}{4}wp - \frac{1}{2}wp}{n} = \frac{1\frac{3}{4}wp}{n}$$

$$= \frac{1.75 \times 16800 \times 15}{8} = 55100$$

Chord BD. c. of m. at c.

$$\frac{1\frac{1}{2}w \times p}{n} = \frac{1\frac{1}{2}\ wp}{n} = \frac{1.5 \times 16800 \times 15}{8} = 47250\ \#$$

Chord Ac. c. of m. at B.

$$\frac{1.5w \times .5p}{n} = \frac{.75\ wp}{h} = \frac{.75 \times 16800 \times 15}{8} = 23625\ \#$$

Diagonals.

Shear in panel EG $= 1\frac{1}{2}w - w - w = -\frac{1}{2}w = -8400$

Stress $= 8400 \sec\theta = 12260 = Fe = -Fg$

Shear in panel CE $= 1\frac{1}{2}w - w = \frac{1}{2}w = 8400$ #

Stress $= 8400 \sec\theta = 12260$ # $= De = -Dc$

Shear in panel AC $= 1\frac{1}{2}w = 25200$ #

Stress $= 25200 \sec\theta = 36800 = Bc = -AB$.

Posts.

Post Cc.

$Dc \sin\theta + Cc + Bc \sin\theta = 0$

$-12260 \sin\theta + Cc + 36800 \sin\theta = 0$

$Cc = -(36800 - 12260) \sin\theta = -24540 \sin\theta = -17850$

Post Ee.

$Fe \sin\theta + Ee + De \sin\theta = 0$

$12260 \sin\theta + Ee + 12260 \sin\theta = 0$

$Ee = -2 \times 12260 \sin\theta = -17850$ #

Live Load Stresses.

Ratio $= \frac{12590}{16800} = .75$

DF = 47200

ce = 41250

BD = 35400

Ac = 17700

$Bc = \frac{(1+2+3)w'}{4} \sec\theta = 6/4 \times 12590 \times 1.46173$

$= 27600$

$AB = -Bc = -27600$

$De = \frac{(1+2)w'}{4} \sec\theta = \frac{3}{4} \times 12590 \times 1.46173 = 13800$

$Dc = -De = -13800$

Cc.

$Dc \sin\theta + Cc + Ec \sin\theta = 0$

$-13800 \sin\theta + Cc + 27600 \sin\theta = 0$

$Cc = -(27600 - 13800) \sin\theta = 10060$ #

Design of Upper Chord.

D.L.	63000
L.L.	47200
Impact L.L. $\frac{300}{300+30}$	42900
	153100

$S = 18000 - 70\ l/r$

Assume section as follows:-

1 Cover plate	12" x 3/8" =	4.5 sq. in.
2 angles 6" x 4" x ½" =		9.50
		14.00 sq.in.

To find the center of gravity of the section take moments about the top.

$$y = \frac{4.5 \times .187 + 9.5 \times 2.365}{14.00} = \frac{.84 + 22.45}{14.00} = 1.675$$

Moments of inertia about the center of gravity:-

$I_a = 1/12 \times 12 \times (.375)^3 + 4.5 \times (1.49)^2 = 10.00$

$I_b = 2 \times 17.4 + 9.50 \times (.315)^2 = \underline{22.80}$

32.80

$r = \sqrt{I/a} = \sqrt{\frac{32.80}{14.00}} = \sqrt{2.34} = 1.53$

$S = 18000 - \frac{70 \times 7.5 \times 12}{1.53}$

$= 18000 - 4120 = 13880$

$\frac{153100}{13880} = 11.03$ sq. in . req'd   O.K.

Rivets req'd $= \frac{14.00 \times 13880}{8000} = 25$ shop or 38 field.

BD.

D.L.	47250
L.L.	35400
Impact	30800
	113450 Total

Use same section as for DF.

Design of End Post AB.

Length = 11' - 0"

Use same section as for DF.

D.L.	36800
L.L.	27600
Impact	23000
	87400

$S = 18000 - 80\ l/r = 18000 - \frac{80 \times 11 \times 12}{1.53}$

$= 18000 - 6900 = 11100$

Area req'd $= \frac{87400}{11100} = 7.87$ Sect. O.K.

25 shop rivets 39 field.

Design of Lower Chord Tension Members.

ce

D.L.	55100
L.L.	41250
Impact $\frac{300}{300 + 75}$	33000
	129350

Net area req'd $= \frac{129350}{16000} = 8.10$

Try 2 angles 6" x 4" x ½" =	9.50
Deduct 2 rivet holes @ .44 =	.88
	8.62 sq. in.

$\frac{8.62 \times 16000}{8000} = 18$ shop rivets or 27 field rivets

Use the same section for the whole lower chord.

Design of Post.

Cc.

D.L.	17850	
L.L.	10060	
Impact L.L. $\frac{300}{300\ \ 45}$	8650	
	36560	Comp.

$S = 16000 - 80\ l/r$

Try 2 angles 4" x 3" x 5/16" Area = 4.18 sq.in.

$I = 2 \times 3.4 = 6.8$

$r = \sqrt{I/a} = \sqrt{\frac{6.8}{4.18}} = 1.28$

$S = 16000 - \frac{80 \times 8 \times 12}{1.28} = 10000$

Area req'd $= \frac{36560}{10000} = 3.66$ sq. in. sect. O.K.

Design of Diagonals.

Bc--Tension member.

D.L.		36800
L.L.		27600
Impact L.L.	$\frac{300}{300 + 45}$	24000
		88400

Net area req'd $= \frac{88400}{16000} = 5.55$ sq.in.

Try 2 angles 4" x 3" x 9/16" Gross area = 7.24

deduct 2 rivet holes @ .49 = .98

6.26

Dc--Compression member.

D.L.		12260
L.L		13800
Impact	$\frac{300}{300 + 45}$	12000
		38060

$S = 16000 - 80\ l/r$

Try 2 angles 4" x 3" x ½" $A = 6.50$

$I = 2 \times 5.0 = 10.0$

$r = \sqrt{I/a} = \sqrt{\frac{10}{6.5}} = \sqrt{1.54} = 1.24$

$S = 16000 - \frac{80 \times 11 \times 12}{1.24} = 16000 - 8520 = 7480$

$\frac{38060}{7480} = 5.08$ sq.in. req'd. O.K.

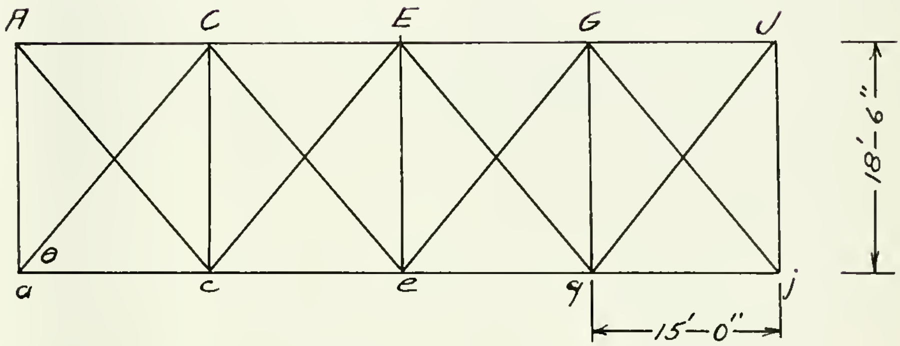
Rivets req'd $= \frac{6.5 \times 7480}{8000} = 7$ shop or 11 field

Lateral Bracing.

D.L. per panel = 4500 same as in previous design.

L.L. per panel = 2250 " " " " " .

Dead Load Stresses.



sec θ = 1.58333

Shear in panel CE = $(2 - \frac{1}{2} - 1)w = \frac{1}{2}w = 2250$

Stress in diagonals = 2250 sec θ = 3560

Shear in panel AC $(2 - \frac{1}{2})w = 1\frac{1}{2}w = 6750$

Stress in diagonals = 6750 sec θ = 10700

Live Load Stresses.

Panel CE. Live load up to e

Stress in diagonals = 1770 sec θ = 28000

Panel AC. Live load up to c.

Shear $= \frac{(1+2+3)w}{4} = 6/4 \times 2250 = 3375$

Stress in diagonals $= 3370$ sec $= 5340$

Stress in Ac.

D.L.	6750
L.L.	5340
	11090

Area req'd $= \frac{11090}{16000} = .695$

Try 1 angle $3\frac{1}{2}$" x 3" x 3/8"   Area=2.3 sq.in.

Net area $= 2.3 - .38 = 1.92$ sq.in.

Shop rivets $= \frac{1.92 \times 16}{8} = 4$ or 6 field rivets .

Design of Reinforced Concrete Abutment.

Design footings and piling for bearing abutment, footings to take all bearing. Pier footings to be set on piling.

Piling:- Safe load per pile $= \frac{2wh}{s + 1}$

w = weight of hammer = 3000#

h = drop of hammer = 20' - 0".

s = penetration of pile for last blow = 1".

Weights and loading:-

Concrete = 150 # per cubic foot.

Weight of bridge determined from design of a 75' Highway Bridge - Class C.

Weight of Live Load - Coopers Specifications.

Bearing Abutments :-

Coarse gravel.

8 tons per square foot allowable.

Loads.

Weight of bridge = 205000 #.

$\frac{205000}{2}$ = 102500 # carried by abutment.

= 51250 # carried by each bridge seat.

Trusses = 18' - 6" center to center.

End span = 60' - 0".

Place traction engine with heavier wheel load directly over the abutment and on one side of the bridge. Then each of the bridge receives 3/4 of the weight of the heavier wheel load plus 3/4 of the reaction of the lighter wheel load.

Heavier wheel = 8 tons = 16000 #

Lighter wheel = 4 tons = 8000 #

3/4(16000 + 50/60 x 8000) = 17000 # for each bridge seat. Distribute load over 12' - 0".

$\frac{17000}{12}$ = 1417 # per lineal foot L.L.

$\frac{51250}{12}$ = 4271 # per lineal foot D.L.

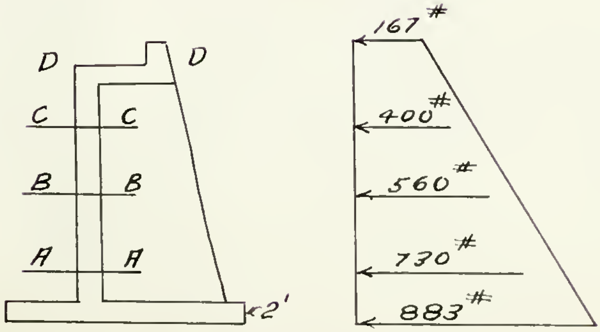
L.L. of 100 # per square foot of remaining floor space gives $\frac{3 \times 100 \times 1.5}{15} + \frac{15 \times 100 \times 7.5}{15}$

= 30 + 750 = 780 # per lineal foot.

D.L. = 4271 # per lineal foot.

L.L. = 1417 780 2197 # per lineal foot.

Total = 4271 + 2197 = 6470 #



$$\text{Surcharge} = \frac{28000 \times 2}{10 \times 12} = 500 \text{ \# per foot}$$

Counterforts:-

$$\text{Pressure at top} = \frac{500}{3} = 167$$

$$\text{Pressure at base} = \frac{500 + 21.5 \times 100}{3} = \frac{2650}{3} = 883$$

$$\text{Resultant} = \frac{167 + 883}{2} \times 21.5 = 11277.5 \text{ \#}$$

Design of section A - A.

Unit pressure at A - A = 730 # per sq.ft.(scaled)

Design as a simple beam 8' span.

$$M = 1/12\ wl^2 = 1/12 \times 730 \times 64 \times 12 = 46720 \text{ "\#}$$

$$d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{46720}{98 \times 12}} = \sqrt{39.7} = 6.3$$

Use a uniform thichness of 12".

$f_s = 15000$ $j = 7/8$ $k = 3/8$ $R = 98$

$$\text{For shear:- Reaction} = \frac{8 \times 730}{2} = 2920 \text{ \#}$$

$$v = \frac{V}{bd}$$

$$d = \frac{V}{bv} = \frac{2920}{12 \times 40} = 6'' \quad \text{O.K. for shear}$$

$$\text{Steel area. } A = \frac{M}{f_s jd} = \frac{46720}{15000 \times 7/8 \times 9} = .390 \text{ Sq.in.}$$

Use $\frac{1}{2}$" ∅ rods spaced 6" centers. A = .390 sq.in.

Section B - B.

8' - 0" above the base.

Unit pressure = 560 # sq.in. (scaled)

$M = 1/12 \times 560 \times 64 \times 12 = 35840$ "#

Steel area = $A = \frac{M}{f_s jd} = \frac{35840}{15000 \times 7/8 \times 9} = .304$ sq.in.

Use $\frac{1}{2}$" Ø rods spaded 9" centers. Area = .33 sq.in.

Section C - C.

13' - 0" above the base

Unit pressure = 400 # per sq.in. (scaled)

$M = 1/12wl^2 = 1/12 \times 400 \times 64 \times 12 = 25600$ "#

Steel Area $A = \frac{M}{f_s jd} = \frac{25600}{15000 \times 7/8 \times 9} = .217$ sq.in.

Use $\frac{1}{2}$" Ø rods spaced 12" centers. Area = .25 sq.in.

Section D - D.

Area Steel req'd = .13 sq.in. (proportionally)

Use $\frac{1}{2}$" Ø rods spaced 12" Area = .25 sq.in,

Steel to tie Slab to Counterfort.

Section A - A.

3' - 0" above the base.

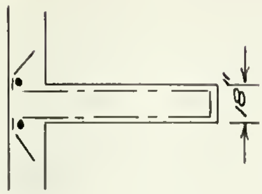
Unit pressure = 730 # per sq.ft.

Reaction of slab at counterfort = 8 x 730 = 5840 #

Steel req'd to tie slab to counterfort equals

$\frac{5840}{15000} = .389$ sq.in.

Bend rods U-shaped as shown in the figure.



$\frac{.389}{2} = .195$ sq.in area of each rod.

Use 3/8" ∅ rods spaced 6" centers.

In order to develop the full strength of the rod, the length must be 62.5 diameters. The length required = 62.5 x 3/8 = 23.5" long. This is for a bond stress of 60 #. O.K. since all rods are longer than 23.5 inches. Length of counterfort at this point = 7' - 8". Extend rods to hook on rods at back of counterfort. Use the same size rods up the entire wall.

Section D - E.

Unit pressure = 560 #

Reaction of slab at counterfort = 8 x 560 # = 4480 #

Steel req'd to tie slab to counterfort = $\frac{4480}{15000}$

= .290 sq.in.

Bend rods to be U-shaped.

$\frac{.290}{2} = .145$ sq.in req'd

Use 3/8 " ∅ rods spaced 8" centers.

Section C - C.

Unit pressure = 400 #

Reaction of slab = 8 x 400 = 3200 #

$\frac{3200}{15000}$ = .214 sq.in steel req'd

Bend rods to be U-shaped.

$\frac{.214}{2}$ = .107 sq.in per rod.

Use 3/8 " ∅ rods spaced 12 " centers.

Vertical Steel in Counterfort.

Total weight of earth above base :-

= $(3\frac{1}{2} \times 16.5 \times 100 + 24.5 \times 7.5 \times 100)8 =$

$(5775 + 18375)8 = 193200$ #

Total area steel req'd $= \frac{193200}{15000} = 12.9$ sq.in to hold up slab.

Use 13 - $\frac{3}{4}$" ∅ rods spaced 6". 13 x 1.04 = 13.52

Area of Steel in Back of Counterfort.

At the base:-

$M = \frac{167 + 830}{2} \times 20.5 \times 6.5 \times 12 \times 8 = 6420000$

$M = f_s Ajd \qquad d = 9.75$

$A = \frac{6420000}{15000 \times .87 \times 9.75 \times 12} = \frac{6420000}{1525000} = 4.22$ sq.in.

Counterfort = 18" 2/3 x 4.22 = 2.81 sq.in for one foot.

Take section 8' - 0" above the base.

$$M = \frac{560 + 167}{2} \times 12.5 \times 5 \times 12 \times 8 = 2185000$$

$d = 7.75$

$$A = \frac{M}{f_s jd} = \frac{2185000}{15000 \times .87 \times 7.75 \times 12} = \frac{2185000}{1215000} = 1.8 \text{ sq.in.}$$

Use 1" ⌀ rods spaced 6" centers $A = 2.00$ sq.in.

Run these rods all the way down to the base.

Section at the base:-

Additional steel req'd:-

$4.22 - 2 = 2.22$ sq.in.

Use 1 1/8" ⌀ rods spaced 6" centers.

Resultant Pressure on Base.

Weight of Counterfort:-

$(3\frac{1}{2} \times 16.5 + 5 \times 9.75)1.5 \times 150 = 24000$ #.

Average weight over 1' of wall $= \frac{24000}{8} = 3000$ #

Resultant acts 7.68 ' from bace of base.

Weight of wall and point of application X.

$$\frac{(3 \times 1.5 \times 150)8.25 + (3.5 \times 1.5 \times 150)10.6 + (1.5 \times 20.5 \times 150)11.75 + (2 \times 17 \times 150)8.5}{\text{Total Weight}} = X$$

$$= \frac{5570 + 7150 + 54250 + 43400}{675 + 675 + 4620 + 5100} = \frac{110370}{11070} = 10.32 \text{ feet.}$$

The point of application of the weight of the abutment is 10.32 feet from the back of the wall.

Total Vertical Force and Point of Application.

$$\frac{(3.5 \times 16.5 \times 100)9.25 + (4.75 \times 24.5 \times 100)3.3 + (11070 \times 10.32) + (3000 \times 7.68) + 6470 + 10.7}{\text{Total Vertical Weight}}$$

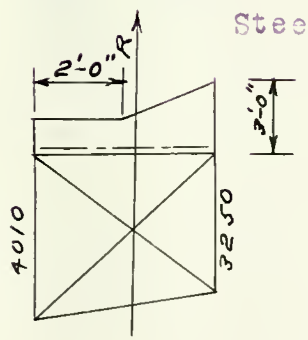
$$= \frac{301790}{37945} = 7.96 \text{ feet.}$$

Horizontal Force = 11277.5 acts 8.8 feet above the base. The resultant falls within the middle third. Therefore O.K.

Eccentricity 2.25 feet.

Pressure Toe $= \frac{37945}{17}\left(1 + \frac{6 \times 2.25}{17}\right) = 4010$ # per sq.ft.

Pressure Heel $\frac{37945}{17}\left(1 - \frac{6 \times 2.25}{17}\right) = 457$ # per sq.ft.



Steel at Toe of Wall.

Shear $= \frac{4010 + 3250}{2} \times 4.5$

$= 16335$ #

Depth req'd for shear equals

$\frac{16335}{12 \times 40} = 35.1$ inches

Make section 3' - 0" and slope to a point 2 ft. from the front.

B.M. $= \frac{4010 + 3250}{2} \times 4.5 \times 2.3 \times 12 = 450846$ "#

$d = 36 - 3 = 33$ ".

$$A = \frac{M}{f_s jd} = \frac{450846}{15000 \times .87 \times 33} = .95 \text{ sq.in.}$$

Use 5/8 " ∅ rods spaced 4" centers.

Steel in Base.

Consider beam from counterfort to counterfort 12" wide. Uniform load = w = 24.5 x 100 = 2450 #

$M = 1/12wl^2 = 1/12 \times 2450 \times 64 \times 12 = 156800$ "#

$$\text{Area steel req'd} = \frac{M}{f_s jd} = \frac{156800}{15000 \times .87 \times 21} = .573 \text{ sq.in.}$$

Use 7/16 " square rods spaced 3" centers for first 8 feet from back of wall.

For next 3½ feet to back of vertical wall:-

w = 16.5 x 100 = 1650 #

$M = 1/12\ wl^2 = 1/12 \times 1650 \times 64 \times 12 = 105500$ "#

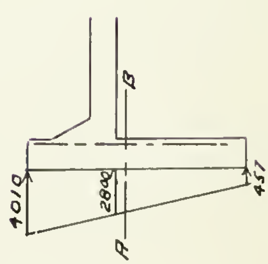
$$\text{Area steel req'd} = \frac{M}{jdf_s} = \frac{105500}{15000 \times .87 \times 21} = .376 \text{ sq.in.}$$

Use 7.16" square rods spaced 6" centers to back of vertical wall.

Rods in Upper Part of Base.

Section A - B.

Unit pressure due to eccentricity of resultant upward on base = 2800 # per square inch.



Unit pressure due to earth above at section (A - B) = 16.5 x 100 = 1650

Diff. = 2800 - 1650 = 1150 acting upward.

$M = 1/12\ wl^2 = 1/12 \times 1150 \times 64 \times 12 = 73600$ "#

Area steel req'd $= \frac{M}{f_s jd} = \frac{73600}{15000 \times .87 \times 21} = .268$

Use 7/16 " Ø rods spaced 6" centers (upper steel)

Steel Intermediate Counterforts.

Section 9' - 0" above the base.

Total horizontal pressure = 1840 #

B.M. = 1840 x 3.5 x 12 x 8 = 618000"#

$A = \frac{M}{f_s jd} = \frac{618000}{15000 \times .87 \times 29} = 1.64$ sq.in.

Use 1" square rods spaced 4" centers.

Area = 3.00 sq.in.

Extend these rods all the way.

Unit pressure at bottom $= \frac{19.5 \times 100}{3} = 650$ #

Therefore total horizontal pressure equals

$\frac{650}{2} \times 19.5 = 6340$ #

B.M. = 6340 x 6.5 x 12 x 8 = 3960000"#

$A = \frac{M}{f_s jd} = \frac{3960000}{15000 \times .87 \times 34} = 8.94$ sq.in. total

8.94 - 3.00 = 5.94 sq.in. more steel req'd.

Use $1\frac{1}{2}$" square rods spaced 4" centers in addition to steel extended down from upper section.

Steel in Bridge Seat.

6470 # per ft. = w = equivalent uniform load.

$M = 1/12\ wl^2 = 1/12 \times 6470 \times 64 \times 12 = 414000$"#

$A = \dfrac{M}{f_s jd} = \dfrac{414000}{15000 \times .87 \times 31} = 1.515$ sq.in. req'd.

Use 7/8 " square rods spaced 6" centers.

Design of Pier and Pier Foundation.

The maximum load on the pier is when the traction engine and the street car are directly over the pier.

(16000 + 50/60 x 8000) = 22670 # traction engine.

(16000 + 50/60 x 16000) = 29350 # street car.

100 # L.L. per sq.ft on remaining surface

= 13500 #

½ D.L. of 75' - 0" bridge = 102500 #

½ D.L. of 60' - 0" bridge = 85500 #

22670 + 29350 + 13500 + 10250 + 85500 = 253520 # is the maximum reaction and load on the pier.

See blue print for the dimensions of the pier.

Weight of concrete = 918750 #

Weight of Base = 211500 #

Total wt. of pier = 1130250 #

Total weight to be held up by pier foundation:-

= 253352 + 1130250 = 1383602 #

Each pile will carry $\frac{2wh}{s+1}$ #

w = weight of hammer = 3000 #

h = drop of hammer = 20' - 0"

s = penetration of last blow = 1"

P = safe load per pile = 2 x 3000 x 20 = 60000 #

No. piles req'd = $\frac{1383002}{60000}$ = 35 piles

Arrangement of piles shown in blue print.

Estimate of Cost of Substructure.

Cubic yards of concrete in piers:-

$V = \frac{h}{2}(a_1 + a_2)$

$a_1 = (6 \times 26.5) + (6 \times \frac{1}{2} \times 3) = 168$ sq.ft.

$a_2 = (9.125 \times 32.75) + (\frac{1}{2} \times 9.125 \times 6) = 325.87$ sq.ft.

$V = \frac{25}{2}(168 + 325.87) = 12.5 \times 493.87$

$= 6160$ cubic feet $= 228$ cubic yards.

Concrete in base:-

$= (3 \times 12.125 \times 35.25) + (\frac{1}{2} \times 3 \times 7 \times 12.125)$

$= 1280 + 127.3 = 1407$ cu.ft. $= 52.2$ cu.yds.

Cubic yards of concrete in one pier:-

$228 + 52.2 = 280.2$

Cubic yards of concrete in two piers:-

$2 \times 280.2 = 560.4$ cubic yards.

The following cost data was found in Gillett's Book on Cost Data. The examples followed were similar to the piers and abutments designed. The conditions under which they were built were also similar to those found in this problem.

Pier supported on Piles.	Cost per cu.yd.
Cement	$ 4.42
Sand	.23
Gravel	.45
Foreman at $ 5.00	.25
Labor at $ 2.00	1.00
Engine man at $ 3.00	.15
Finish coat at $3.00	.06
Carpenters at $ 3.00	.34
Forms at $ 23.50, used once	.80
Wire, nails etc	.02
Pro rated plant cost	.53
Coffer dams	1.60
Coffer dam excavation	.34
Total	$ 10.09

560.4 x 10.09 = $ 5654.44 Cost of two piers.

Cubic Yards of Concrete in Abutments:-

Base $= \left(2 \times \frac{23 + 20}{2} \times 17\right) + \left(2 \times \frac{17 \quad 6.25}{2} \times 44 \times 2\right)$

= 2760 cubic feet = 102.2 cubic yards.

Extra section at base:-

$(\frac{1}{2} \times 1 \times 2.25 \times 106) = 129.25$ cu.ft = 4.8 cu.yds.

Vertical wall = (1 x 16.5 x 104) = 1716 cubic feet.

= 63.6 cubic yards.

Seat $= (1\frac{1}{2} \times \frac{14+10}{2} \times 3.75) = 67.5$ cubic feet

$= 2.5$ cubic yards.

Parapet $= (1\frac{1}{2} \times 3 \times \frac{28+20}{2}) = 108$ cu.ft $= 4$ cu.yds.

Counterforts:-

$2 \times \frac{9+5}{2} \times 16.5 \times 1\frac{1}{2} = 346$ cu.ft. $= 12.8$ cu.yds.

$2 \times \frac{7.5+2}{2} \times 16.5 \times 1\frac{1}{2} = 235$ cu.ft. $= 8.7$ cu.yds.

$2 \times 6/2 \times 15.5 \times 1\frac{1}{2} = 139.5$ cu.ft. $= 5.2$ cu.yds.

$2 \times 4/2 \times 12 \times 1\frac{1}{2} = 72$ cu.ft. $= 2.7$ cu.yds.

$2 \times 2.5 \times 8.5 \times 1\frac{1}{2} = 15.9$ cu.ft $= .6$ cu.yds.

Total $= 102.2 + 4.8 + 63.6 + 2.5 + 4.0 + 12.8 + 8.7 + 5.2 + 2.7 + .6 = 207.1$ cubic yards in one abutment.

$2 \times 207.1 = 414.2$ cubic yards of concrete in the two abutments.

Weight of Steel Reinforcing.

In figuring the weight of the steel reinforcing the lengths of the rods were scaled off the blue print and multiplied by the weight per foot of rod as given in the hand book of the Carnegie Steel Company.

The following weights are for one half of the abutment.

Base - Lower steel	32 x 30 x .651 =	625.00 #
" - " "	50 x 14 x .651 =	455.70 #
" - Upper "	32 x 15 x .511 =	245.00 #
" - " "	50 x 14 x .511 =	357.70 #
" - Trans. "	331 x 1.043 =	345.00 #

Counterforts:-

Vert. steel	6 x 65 x 1.502 =	585.00 #
At Back	15 x 20 x 3.4 =	1020.00 #
" "	9 x 20 x 3.4 =	612.00 #
Hor. steel	5 x 100 x .376 =	188.00 #

Wall :-

Vert. steel	22 x 17 x .668 =	250.00 #
Hor. steel	27 x 37 x .668 =	660.00 #

Seat and parapet = 12.5 x 10 x 2.603 = 325.00 #

Total weight of one half of the abutment equals 5668.40 # weight of steel.

2 x 5668.40 = 11336.8 # weight of steel in one abutment.

2 x 11336.8 = 22673.6 # weight of steel in two abutments.

Cost of Abutments.

Material	Cost per cu.yd.
Cement @ \$ 3.73	\$ 4.42
Sand @ \$.50½	.23
Gravel @ .50½	.45
Lumber @ \$ 23.33	.88
Piles @ \$.22	.09
Machinery	.10
Wire and nails	.18
Lubricating oil	.01
Fuel	.18
	\$6.54

Labor	Cost per cu.yd.
Excavation for foundation	\$ .34
Building and removing forms	.57
Driving piles in foundation	.11
Placing steel reinforcement	.16
Mixing concrete	.38
Placing concrete	.17
Pumping water	.03
Cleaning and storing machines	.10
	\$ 1.86
	6.54
Total material & labor	\$ 8.40

Cost of steel reinforcing at $ .04 per pound

= 22673.6 x .04 = $ 906.94

Total cost of abutments = 3479.28 + 906.94 =

$ 4386.12

Total cost of piers $ 5654.44

Total cost of piers and abutments $ 10040.56

Cost of Piles.

Cost of piles delivered at $ .25 per foot.

= 52 x 10 x .25 = $ 130.00

Cost of driving piles at $ .14 per foot.

= 52 x 10 x 14 = $ 72.80

Total coat of piles $ 202.80

Total cost of substructure:-

$10040.56 + $ 202.80 = $10243.36

Estimate of Cost of Superstructure.

Weight of Steel in 75'- 0" Span.

The following weights are for one half of one truss.

Member.	Length	Weight.
Upper Chord.		
2 angles 6" x 4" x 5/8"	30'-0"	1200.00 #
1 Cov. Pl. 12" x 5/8"	30'-0"	612.00
1 Splice Pl. 12" x 5/8"	3'-0"	76.50
1 Splice Pl. 12" x ½"	18-0"	20.40
End Post.		
2 angles 6" x 4" x 5/8"	10'-0"	400.00
1 Cov. Pl. 12" x ½"	10'-0"	204.00
Lower Chord.		
2 angles 6" x 4" x 7/8"	37'-6"	2040.00
1 Splice Pl. 12" x 3/8"	0'-10"	12.75
1 Splice Pl. 12" x 3/8"	3'-1"	47.17
1 Splice Pl. 12" x 3/8"	2'-6"	37.80
Verticals.		
4 angles 4" x 3" x 5/16"	6'-6"	187.20
4 Batten Pl. 12" x ½"	0'-5"	34.00
4 Ext. Ang. 4" x 3" x5/16"	3'-3"	93.60

Diagonals	Length	Weight.
2 angles 4" x 3" x 11/16"	9'-0"	260.40 #
4 angles 4" x $3\frac{1}{2}$" x $\frac{3}{4}$"	9'-0"	626.40
3 angles 3" x 3" x $\frac{1}{2}$"	9'-0"	169.20
4 clip ang. 4" x $3\frac{1}{2}$" x $\frac{3}{4}$"	1'-0"	69.60
4 batten pl. 12" x $\frac{1}{4}$"	0'-5"	17.00
4 batten pl. 12" x $\frac{1}{4}$"	0'-10"	34.00
Gusset Plates.		
2 Plates $\frac{1}{2}$" x 1'-10"	2'-3"	162.30
2 Plates $\frac{1}{2}$" x 2'-0"	4'-1"	333.20
2 Plates $\frac{1}{2}$" x 1'-8"	3'-2"	215.30
3 Plates $\frac{1}{2}$" x 1'-1"	1'-5"	40.50
2 Plates $\frac{1}{2}$" x 1'-10"	3'-3"	343.10
2 Plates $\frac{1}{2}$" x 1'-7"	2'-7"	166.90
Shoe.		
2 angles 7" x $3\frac{1}{2}$" x $\frac{1}{2}$"	1'-0"	34.00
2 plates $\frac{1}{2}$" x 12"	1'-9"	71.40
Lacing.		
65 @ $1\frac{1}{2}$" x 3/16"	2'-2"	130.06
25 @ 2" x $\frac{1}{4}$"	1'-2"	55.42
16 @ $1\frac{3}{4}$" x $\frac{1}{4}$"	1'-2"	25.05
70 @ $1\frac{3}{4}$" x $\frac{1}{4}$"	1'-0"	104.30

The following weights are for one half of the bridge.

Lower laterals	Length	Weight.
1 angle $2\frac{1}{2}$" x $2\frac{1}{2}$" x $\frac{1}{4}$"	19'-6"	80.00 #
18 seat ang. $2\frac{1}{2}$" x 2" x $\frac{1}{4}$"	0'-7"	38.01
5 angles $3\frac{1}{2}$" x 3" x 3/8"	16'-0"	632.00
Stringers.		
7-12" I-beams @ 27.5 #	38'-0"	7315.00
2-12" I-beams @ 31.5 #	38'-0"	2393.00
$2\frac{1}{2}$ gusset pl. 3/8" x 11"	2'-6"	89.50
Buckle Plates.		
600 sq. ft. @ 10.20 # per sq.ft.		6120.00
Rivets.		
2574 x 2 @ 16 # per 100 heads		823.00

The total of the weights as given for one half of one truss equals 7844.55 #. The total weight of these members in the bridge equals
4 x 7844.55 = 31378.20 #

The total of the weights as given for one half of the bridge equals 17490.51 #. The total weight of these members in the bridge equals
2 x 17490.51 = 34981.02 #

The total weight of steel in the bridge equals 31378.20 + 34981.00 = 66359.20 # . The weight of the details of this bridge equals 4 x 1661.05 = 6644.20 . The percent of the details = $\frac{6644.20}{66359.20}$ = 10 %

Weight of 60'-0" Span.

Upper Chord	Length	Weight.
2 angles 6" x 4" x ½"	22'-6"	729.00 #
1 Cov. Pl. 12" x 3/8"	22'-6"	336.00
End Post.		
2 angles 6" x 4" x ½"	10'-6"	340.20
1 Cov. Pl. 12" x 3/8"	10'-6"	160.15
Verticals.		
2 angles 4" x 3" x 5/16"	6'-6"	93.60
Lower Chord.		
2 angles 6" x 4" x ½"	30'-0"	972.00
Diagonals.		
2 angles 4" x 3" x 9/16"	9'-0"	223.20
4 angles 4" x 3" x ½"	9'-0"	399.60
Lower laterals.		
2 angles 3½" x 3" x 3/8"	10'-0"	252.80
9 Seat ang. 2½" x 2" x ¼"	0'-9"	24.44
1 angle 2½" x 2½" x ¼"	9'-9"	40.00

Stringers.	Length	Weight.
$3\frac{1}{2}$ - 12" I-beams @ 27.5	30'-0"	2887.50 #
1 - 12" I-beam @ 31.5	30'-0"	945.00
2 Gusset Pl. 3/8" x 11"	2'-6"	71.50
Buckle Plates.		
240 sq. ft. @ 10.20 # per sq.ft. =		2448.00

The weight of one half of one truss equals 9923 #. The weight of the bridge, without the details equals 4 x 9923 = 29792 #. Add 10 % for details. 39792 + 2979.2 = 32771.00 # total weight of the bridge.

There are two bridges having a span equal to 60'-0" so therefore the weight of the two bridges will be 2 x 32771 = 65542 #. The total weight of steel in the superstructure equals 65542 + 66359 = 131901 #.

The cost of the bridges is $ .04 per pound of steel. 131901 x .04 = $ 5276.04 . This includes the cost of material, construction and painting.

As the bridge will be built in the same city as it is to be erected in, the haul will be short. The total transportation will cost $ 50.00 making a total of $ 5326.00

Cost of Asphalt Pavement.

There are 156 square yards of pavement to make on the three bridges and the two approaches. The average price per square yard of asphalt pavement is \$ 2.05. This is for a pavement guarantied for five years, with a 4" concrete base. 156 x 2.05=\$319.80 total cost of pavement.

Cost od Approaches.

The two approaches will be dirt fills with slopes of 1½ to 1. The total number of cubic yards in the two fills is 1157. There is no over haul and therefore the fill will cost \$ .17 per cubic yard. 1157 x .17 = \$ 196.70. It will be necessary to have a wire fence along the approach, the total length being about 200 feet. Use 5" cedar posts set 30" in the dirt with the wire at 15" centers. This will cost about \$ 5.00.

Total cost of superstructure=\$ 6847.54
Total cost of substructure= 10243.36
\$17090.90

The total cost is \$17100.00 .

Bibliography.

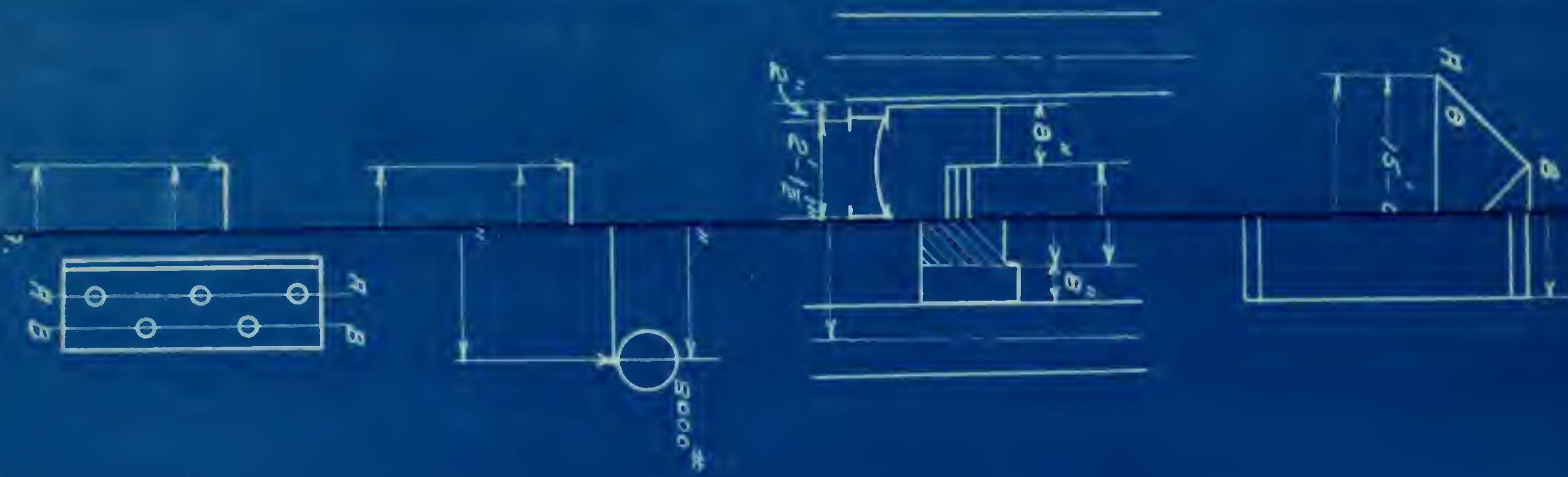
Wells. Steel Bridge Designing.

Turneaure and Maurer. Principles of Reinforced Concrete Construction.

Gillette. Cost Data.

Hand Book of Carnegie Steel Company.

Hand Book of Cambria Steel Company.



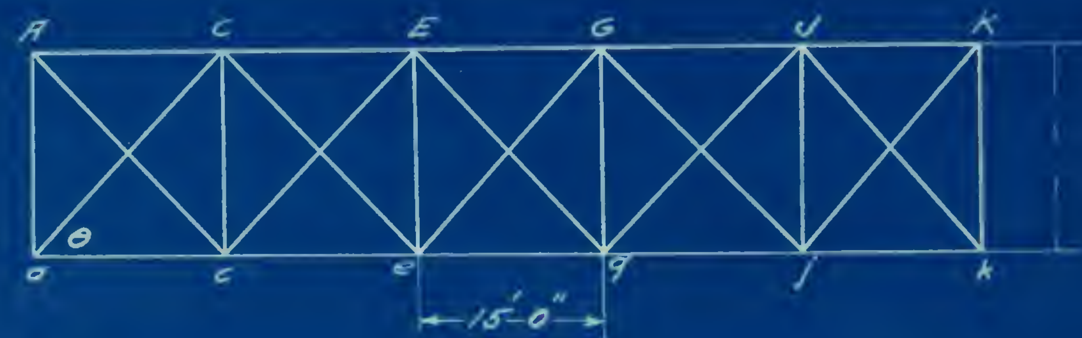


Fig. 9.

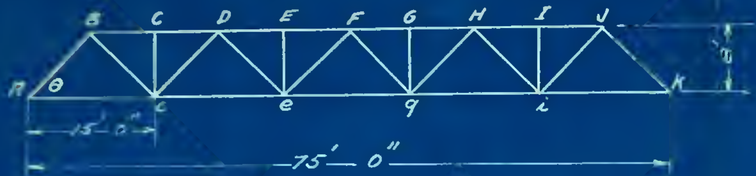


Fig. 1.

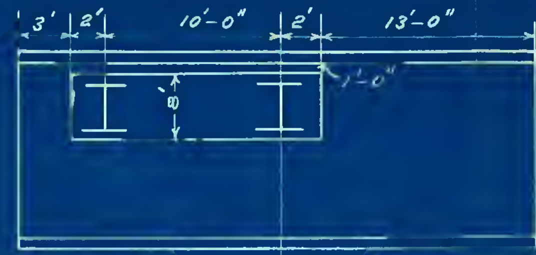


Fig. 6 a.

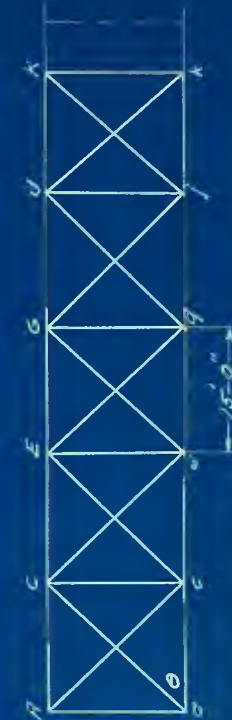


Fig. 9.

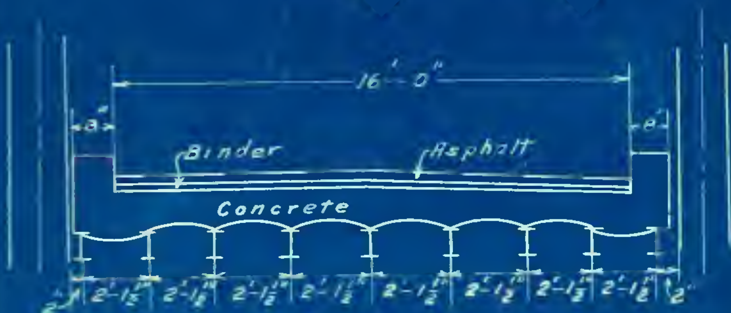


Fig. 2.

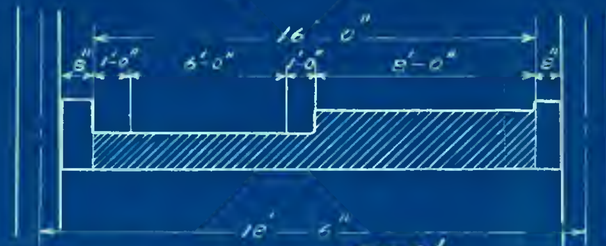


Fig. 6 b.

Fig. 3.

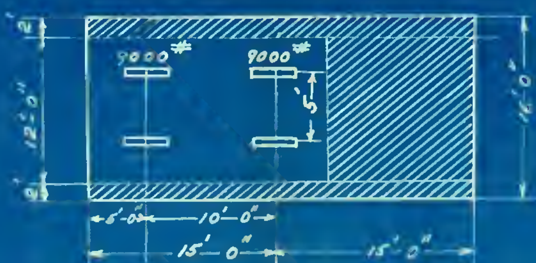


Fig. 5.

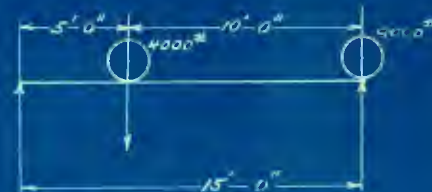


Fig. 7.

Fig. 4.

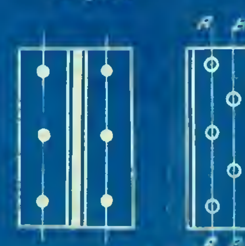
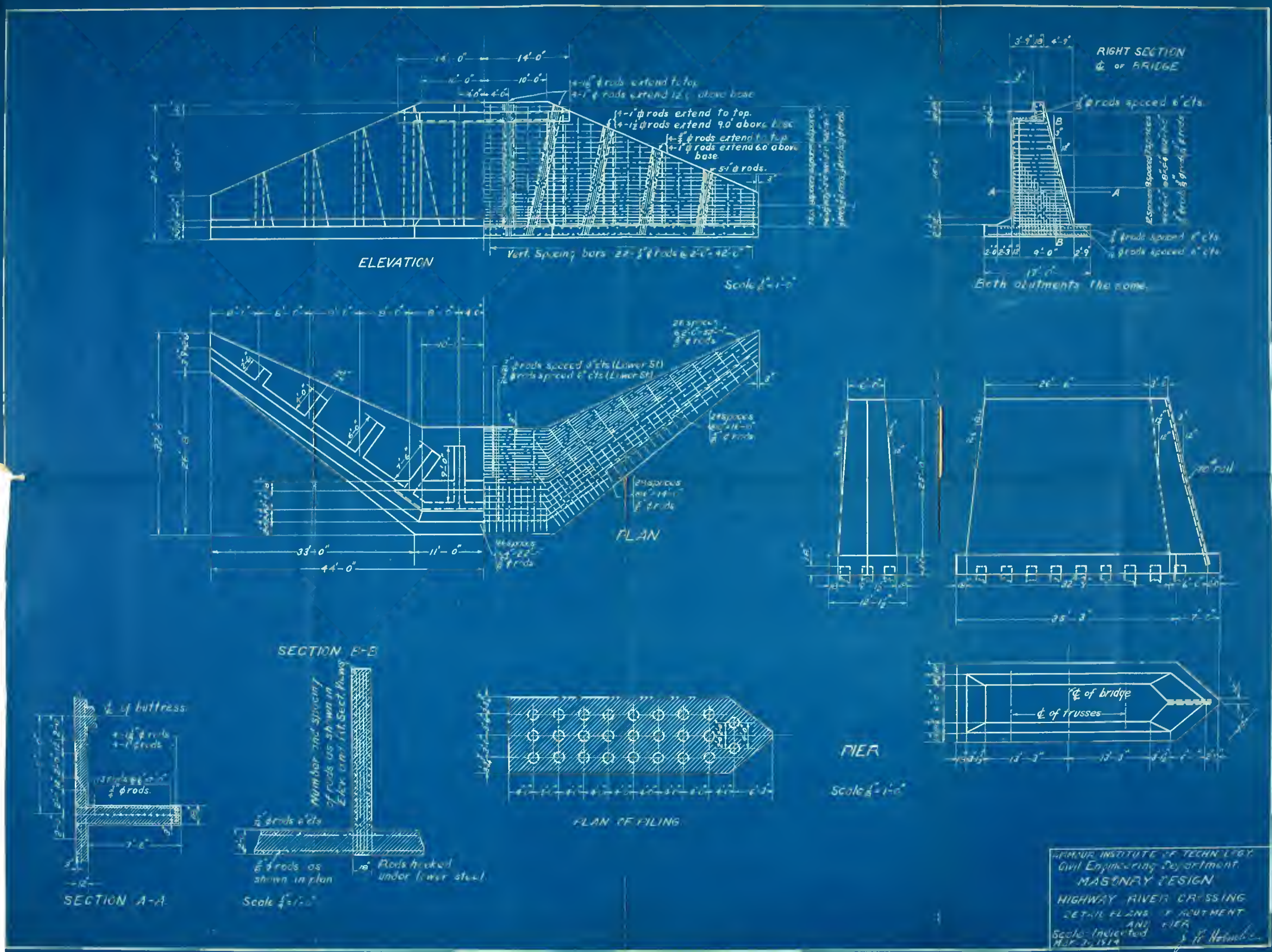
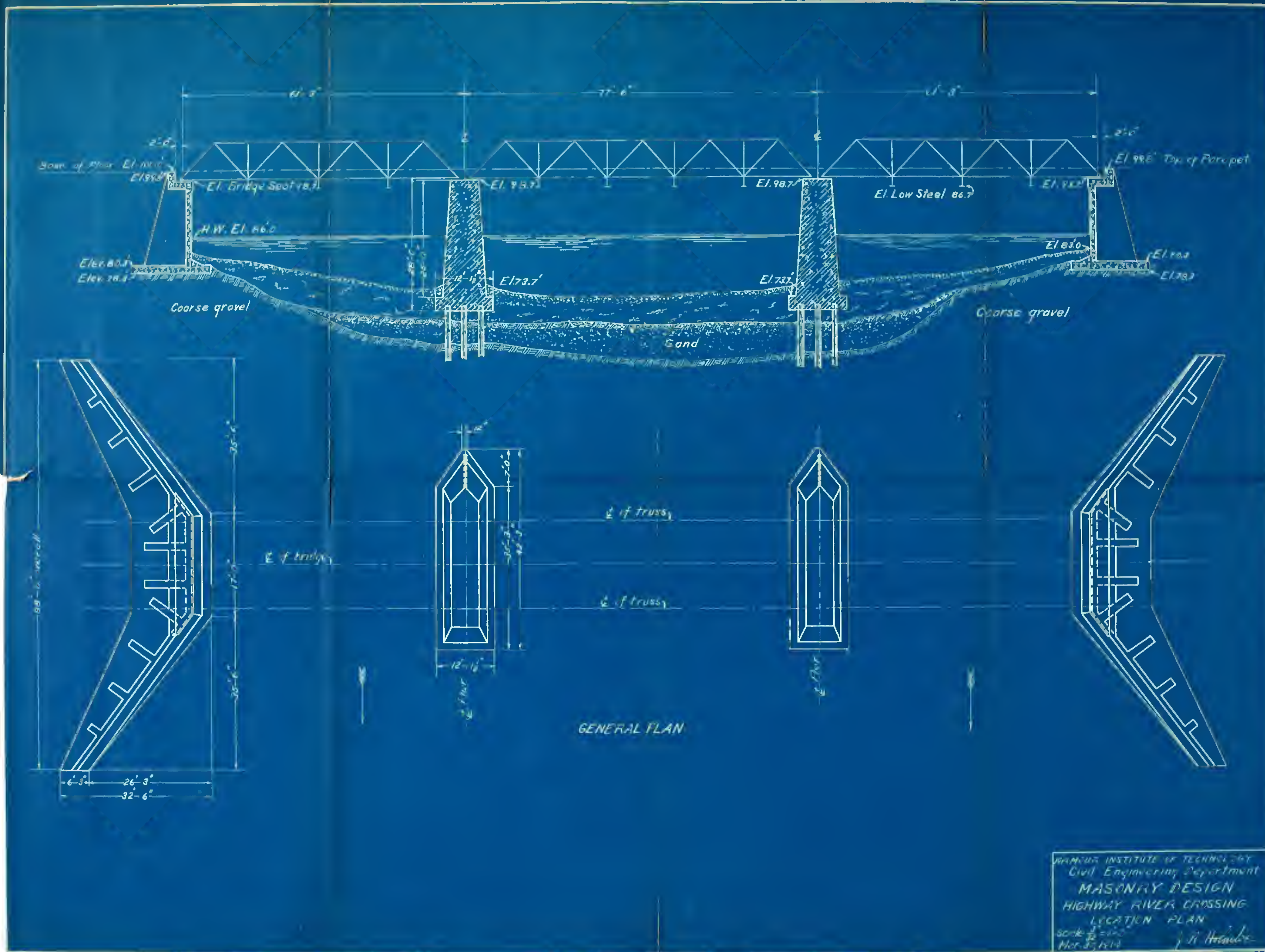
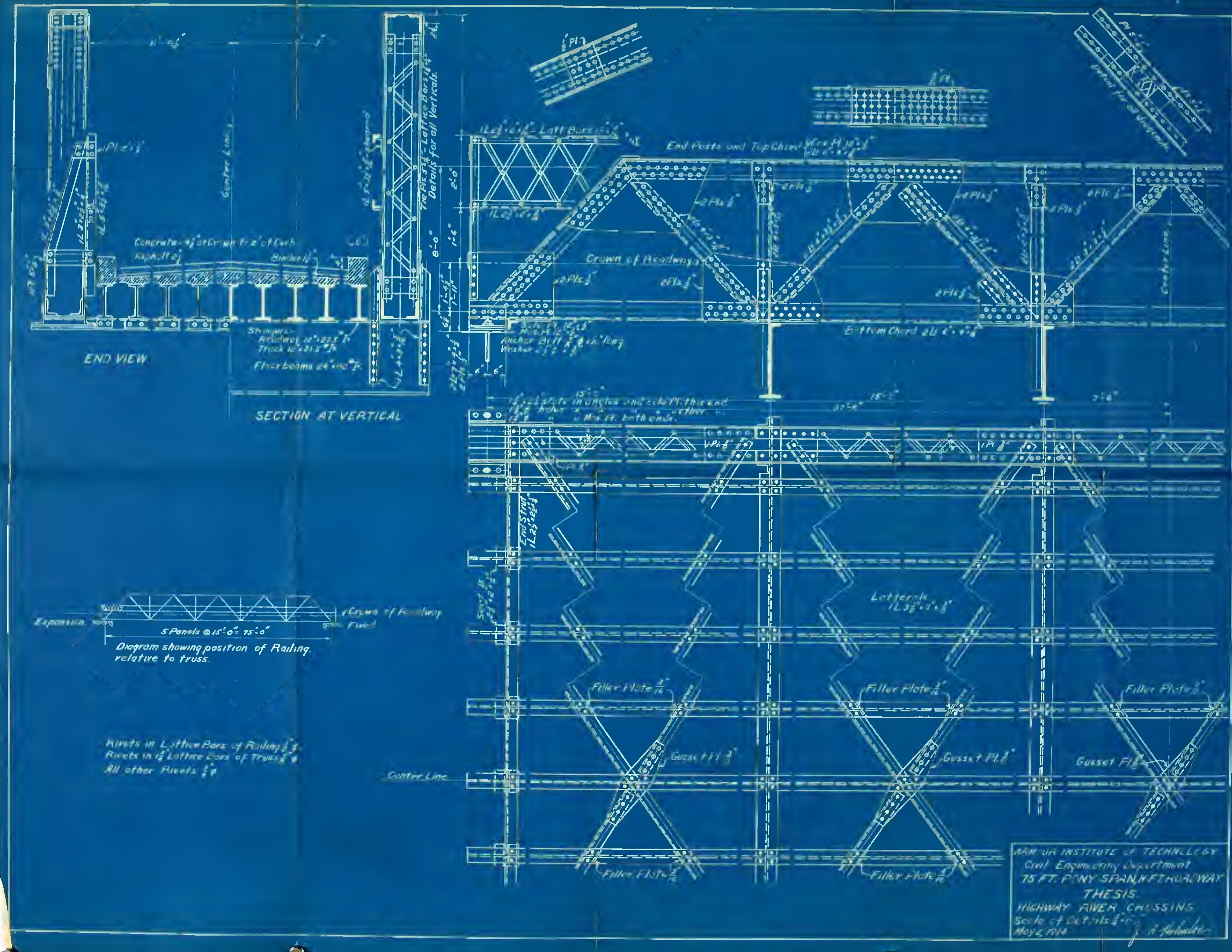


Fig. 8.


ELEVATION
Vert. Spacing bars
Scale
RIGHT SECTION
℄ of BRIDGE
Both abutments the same.
PLAN
PIER
Scale
SECTION B-B
SECTION A-A
℄ of buttress
Number and spacing of rods as shown in Elev. and Rt. Sect. Dwg.
Rods hooked under lower steel
FLAN OF PILING
℄ of bridge
℄ of trusses
ARMOUR INSTITUTE OF TECHNOLOGY
Civil Engineering Department
MASONRY DESIGN
HIGHWAY RIVER CROSSING
DETAIL PLANS OF ABUTMENT AND PIER
Scale: Indicated


77'- 6"
2'-6"
El. 99.6' Top of Parapet
El. Bridge Seat 98.7
El. 98.7
El. 98.7
El. Low Steel 86.7
H.W. El. 86.0
El. 83.0
Elev. 80.3
Elev. 78.3
El. 73.7'
El. 73.7
Coarse gravel
Coarse gravel
Sand
℄ of truss
℄ of bridge
℄ of truss
℄ Pier
℄ Pier
6'-3"
26'-3"
32'-6"
35'-6"
17'-0"
35'-6"
7'-0"
12'-1½"
GENERAL PLAN
ARMOUR INSTITUTE OF TECHNOLOGY
Civil Engineering Department
MASONRY DESIGN
HIGHWAY RIVER CROSSING
LOCATION PLAN

Holotype.


END VIEW
SECTION AT VERTICAL
Center Line
Concrete
Asphalt
Binder
Stringers-
Floorbeams 24"
End Posts and Top Chord
Crown of Roadway
Bottom Chord
End Strut
Laterals
Filler Plate
Gusset Pl.
Center Line
Details for all Verticals.
Detail for all Diagonals
5 Panels @ 15'-0" = 75'-0"
Expansion
Fixed
Crown of Roadway
Diagram showing position of Railing relative to truss.
Rivets in Lattice Bars of Railing
Rivets in Lattice Bars of Truss
All other Rivets
ARMOUR INSTITUTE OF TECHNOLOGY
Civil Engineering Department
75 FT. PONY SPAN, ROADWAY
THESIS.
HIGHWAY RIVER CROSSING
Scale of Details
May 1914